- Evaluation of the alternative with regard to the water quality criteria was based on computation residence time for each of the alternative.
- Results of computation show that most of jetty extension, including Alternative 3 may increase slightly the residence time. It implies on slight deterioration of the water quality. Observation on the model shows that quality of the water in the harbor (residence time) is inversely proportional to blockage by the jetty the upper water column area.
- Alternative 3B would reduce slightly the residence time in the harbor. This effect is due to the fact that the jetty extension is submerged. Lowering the jetty crest elevation would reduce residence time and improve the water quality in the harbor if requires. reduction of water
- Alternative 5, Jetty Relocation would reduce the residence time and most likely improve the water quality in the harbor. .

5. Keystone Physical Modeling

5.1. Objectives of Physical Modeling

Physical modeling of the Keystone Ferry Terminal and the jetty extension alternatives was conducted to meet the following goals:

- Verify and validate the numerical modeling of tidal flow circulation and wave transformation
- Qualitatively evaluate the alternatives with regard to their effect on shoreline changes in the vicinity of the jetty
- Quantitatively assess the effects of the jetty extensions on cross-channel currents at the entrance to Keystone Harbor

5.2. Modeling Facilities

The physical modeling was conducted in the three-dimensional wave basin at the O.H. Hinsdale Wave Research Laboratory (WRL) at Oregon State University. The basin is one of the largest facilities in the United States that equipped with most modern technologies and equipment to provide modeling of coastal processes in a high resolution scales. Tank dimensions are as follows. Length - 160 ft, width - 87 ft, depth - 7 ft. The picture of wave tank facilities during construction of the model is shown in Figure 5.1.

Figure 5.1 WRL modeling facility during construction of the model

The tank has been equipped with wavemaker and current generated system. The wavemaker is a multi-panel piston-type with a programmable actuator and active wave absorption capable of generating complicated and realistic waves with irregular directional spectra. The current generated system was built to simulate Keystone Harbor entrance cross current velocities and consists of a pumping system, outflow manifold, and sump. The sump system uses approximately 20% of the model basin.

The wave basin is traversed by a movable bridge which is used to mount and position data acquisition gages. Also, a heavy-duty crane is available for heavy moving if required.

Figure 5.2 shows a schematic of the model layout in the WRL 3D wave basin. The wavemaker is positioned at the bottom of the figure and can generate waves from approximately ± 15° from normal to the wavemaker.

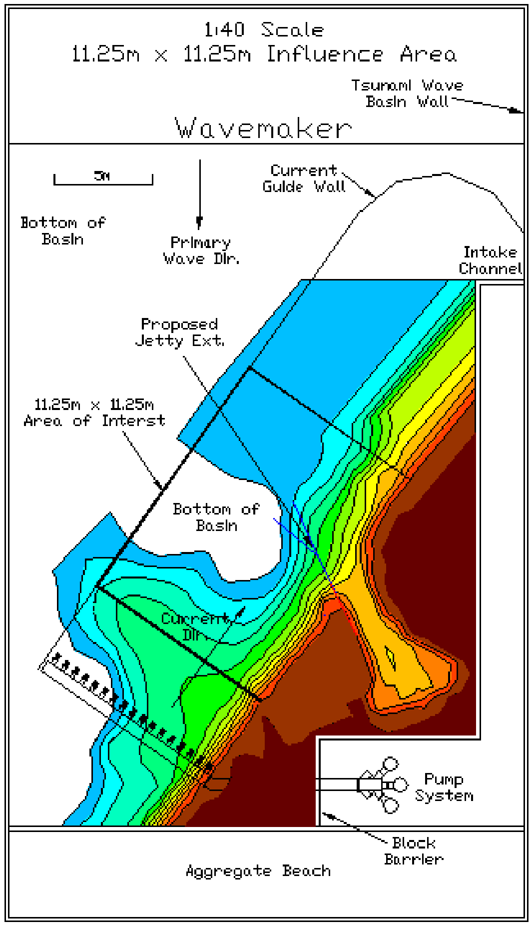

Figure 5.2 1:40 Scale model setup in 3D wave basin at the OSU WRL.

5.3. Model Setup and Modeling Methodology

This section describes the scaling and design of the physical model. It also discusses generation and measurement of the waves and current as well as the measurement of the shoreline changes.

5.3.1. Model Scaling

The physical model was designed and optimized to provide maximum resolution for modeling physical processes and satisfy similarity parameters while considering limitations of WRL facilities. Model scaling was conducted using proved modeling methodology (Table 5.1) and advanced technologies available in the industry.

The model's dominant processes are related to free-surface flows; therefore, Froude scaling is used to ensure similitude between the coastal processes and physical parameters of the model and prototype[9]. Two types of sediment were introduced to the model and were studies in the model: sand and large gravel and cobble material. Considering the uncertainties with the scaling for fine sediment (See footnote) the results of physical modeling that include the sediment are considered qualitative only can are used herein in relative comparison analysis only.

In order to balance the physical limitations (water depth, model extents, jetty stone size, available sediment for beach material) and required magnitude of the coastal processes (wave height, period, current velocity), a scale factor of 1M:40P (1:40 or scale factor $sf = 40$) was selected. Some of the dimensions of various model parameters are shown in prototype and model scale in Table 5.2. Note that the d_{50} grain sizes of sediments are scaled so that the fall velocity of the prototype sediments matches the scaled fall velocity of the model sediments; they are not scaled via the length scale (Hughes, 1993).

Table 5.1 Physical Model Scaling Factors

$$\begin{aligned} &\text{Length Scale:} && L_p = sf * L_m \\ &\text{Time Scale:} && T_p = \sqrt{sf} * T_m \qquad (1) \\ &\text{Velocity Scale:} && V_p = V_m * \sqrt{sf} \end{aligned}$$

Table 5.2 Prototype and Model dimensions based on 1:40 Froude scaling.

Parameter	Prototype		1:40 Model	
Depth, h	25	ft	0.63	Ft
H_{min}	3	ft	0.08	Ft
H_{max}	4	ft	0.10	Ft
T_{min}	4	sec	0.63	Sec
T_{max}	5	sec	0.79	Sec
Approx Jetty Length	500	ft	12.50	Ft
Jetty Entrance Width	250	ft	6.25	Ft
d_{50} sand	0.3	mm	0.095	Mm
d_{50} cobble	3.0	cm	1.05	Mm
Peak current	1.2	m/s	0.19	m/s

[9] A fundamental difficulty exists in modeling free-surface processes if the same model is assigned to simulate sediment transport processes. Waves and tidal currents are dominated by free-surface (thus, Froude) processes. However, sediment transport (if fine sediment, including sand and small gravel) is primarily controlled by processes described by Reynolds similitude. It is not feasible to accurately scale physical models to match both Reynolds and Froude similitude without using a fluid with different properties (i.e., viscosity and density) than water. For models of large scale – such as this model - it is not practical to use a fluid other than fresh water; therefore, a distortion in the scale effect of the transport processes occurs. For more information on this distortion, refer to Hughes (1993).

5.4. Model Design

5.4.1. Existing Conditions

The existing conditions of the project site were first constructed and tested to establish the baseline existing conditions at the project site for comparison to the changes induced by the various alternatives. Figure 5.3 shows a schematic of the layout of the existing conditions of the model in the 3D wave basin, and Figure 5.4 shows a photograph of the model of existing conditions in the wave basin. The existing topography and bathymetry at the project was based on the most recent survey data available. The topographic and bathymetric contours were constructed by first cutting shore-parallel cross-sections every 2m across the model. These cross-sections were cut from plywood and placed in the basin. Next, the area between the cross-sections was filled with sand and aggregate to match the elevation along the cross-sections, see Figure 5.5. Finally, the area was covered with a thin layer (approximately 3.175 cm) of concrete which was smoothed to best represent the existing contours. A survey, performed by Epic Scan using LIDAR (LIght Detection And Ranging), of the resulting model bathymetry is shown in Figure 5.6. The difference between actual bathymetry data and model survey is shown in Figure 5.7 in prototype scale. Comparison of the bathymetry and model survey data shows that the model is constructed with appropriate accuracy and is adequate to conduct modeling in scale 40:1.

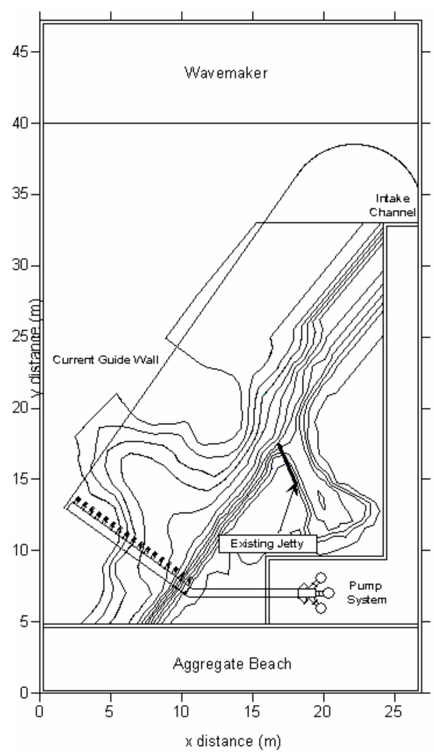

Figure 5.3 Layout for existing conditions in 3D wave basin

Figure 5.4 Photo of model of existing conditions

Figure 5.5 Photo of model topography and bathymetry during construction

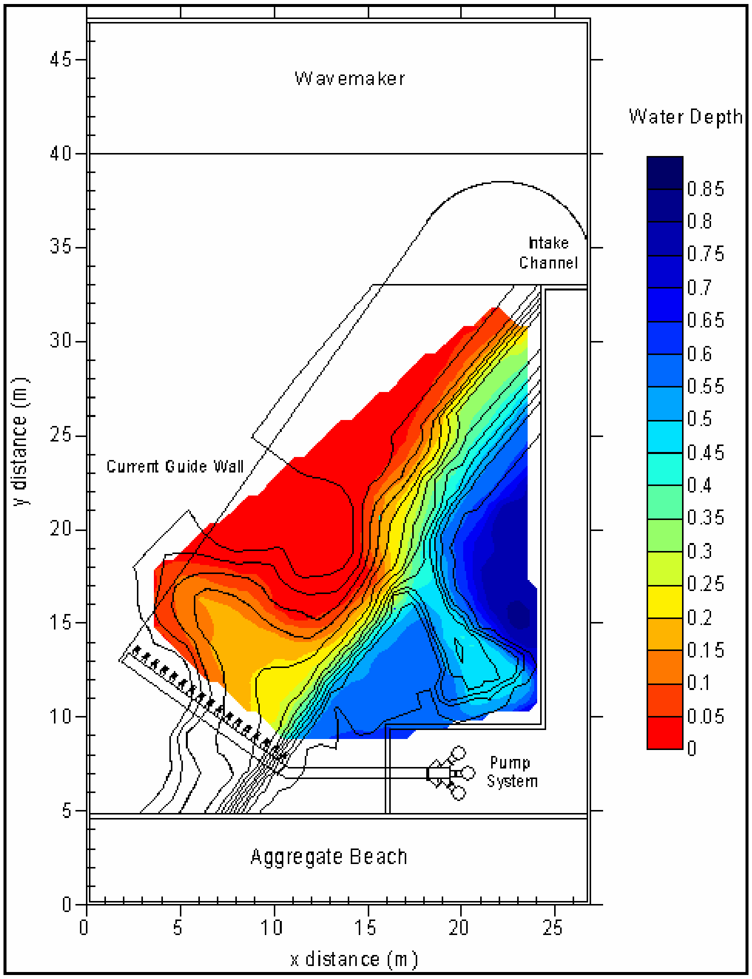

Figure 5.6 Contour of the physical model in the wave basin (based on LIDAR data).

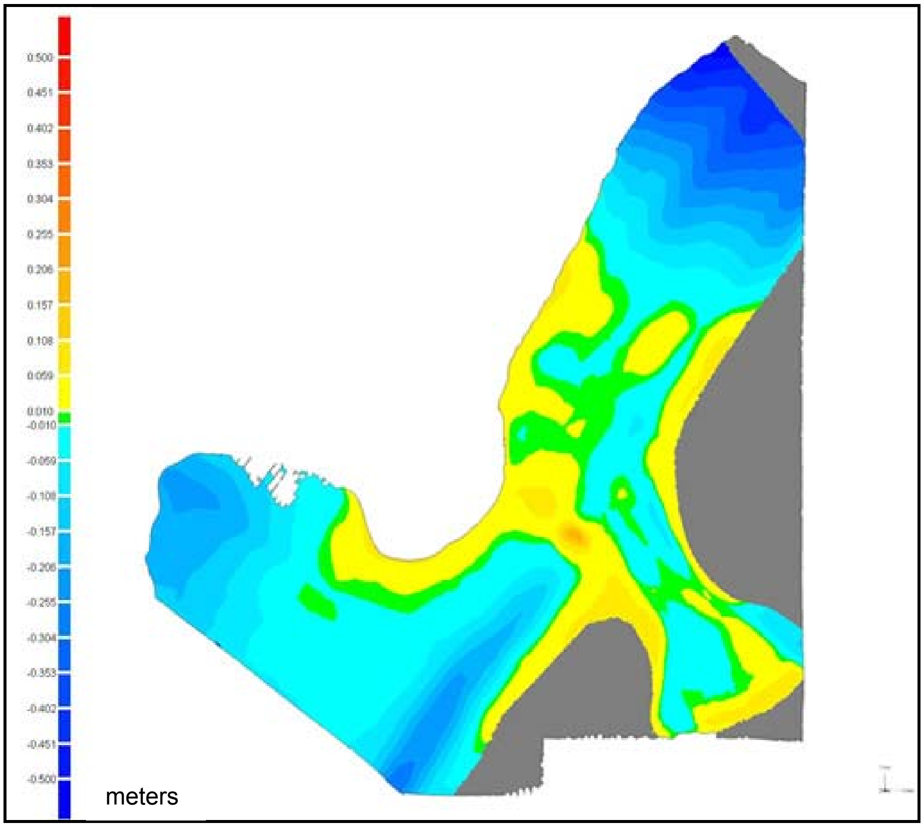

Figure 5.7 Bathymetry difference, between prototype and model in prototype scale

The existing jetty was constructed out of 54.5 mm (model-scale) rock. The modeled jetty rock was scaled to assure stability of the rock and jetty integrity for all modeling wave and current scenarios. The jetty was constructed to match the existing conditions of 386 ft in length above SWL, an 8 ft crest width, and 1.25:1 side slopes. Figure 5.8 shows a photograph of the model of the existing jetty. It should be noted that SWL in the physical model corresponds to MLLW tide elevation at the prototype.

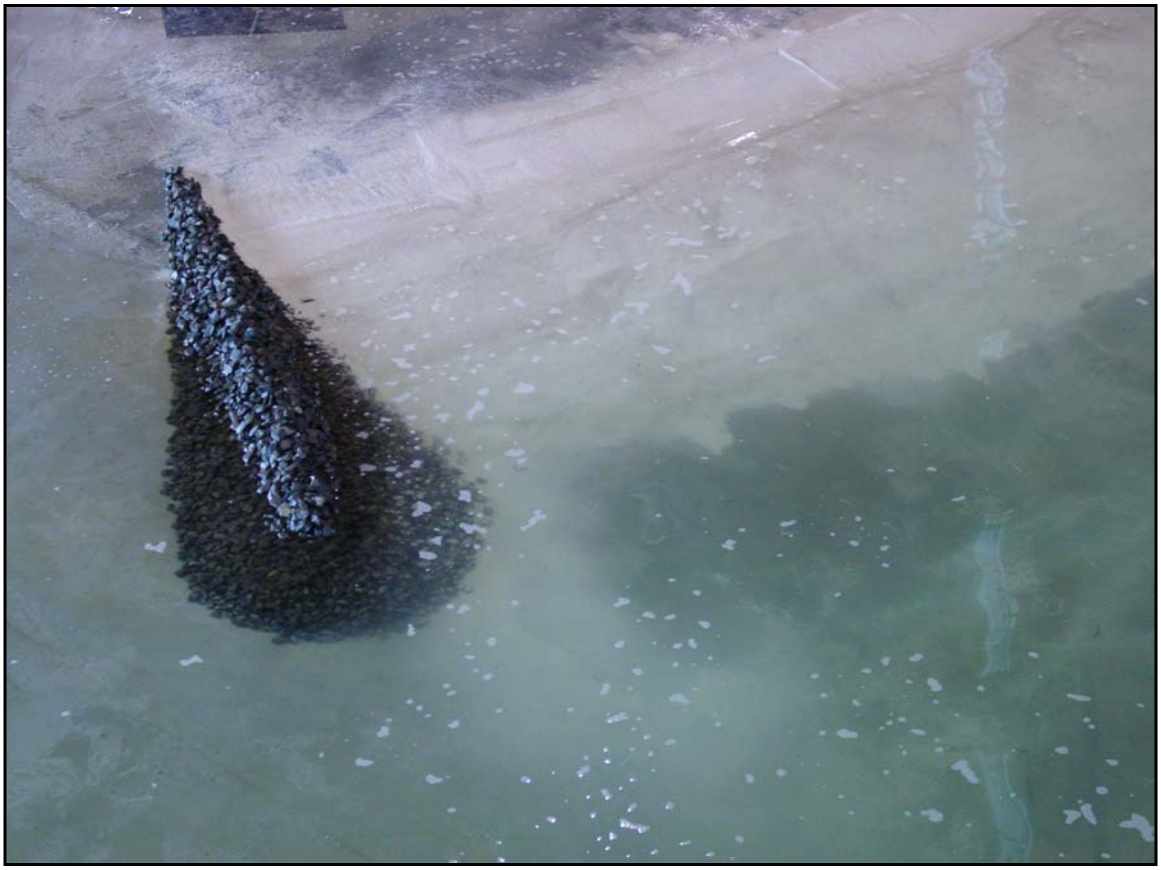

Figure 5.8 Modeled existing conditions

To evaluate changes in the shoreline to the east of the jetty, the sediment was placed on the model to the east from the jetty at approximate length 13.1 ft (524 ft prototype). Based on existing conditions observed during site visits, the sediment at the east side of the jetty contained a mixture (See Figure 5.9) consisting of cobble with a mean grain size diameter of approximately d_{50} = 3cm (prototype) and sand with a mean grain size diameter of d_{50} = 0.3mm (prototype). To model this mixture, two types of material were used. The cobble was modeled with "aquarium sand" (also known as 16 mesh grain size), which has a d50 = 1.0mm, and the sand was modeled with "fine silica sand" (also known as 70 mesh grain size) sand that has a d50 = 0.095mm. The 1.0mm cobble scales up to approximately 3cm in prototype, and the 0.095mm sand scales to approximately 1.0mm sand in prototype. Figure 5.10 shows the results of grain size sieve analyses performed on different types of material available for WRL and comparison them to two types of prototype material (dashed lines on the graph). The cobble and sand were mixed at a 1:1 ratio to best match conditions at the project site. Approximately 0.68 cy of material was placed to the east of the jetty in the model. Figure 5.11 is a photograph of the sediment mixture in place in the model.

Figure 5.9 Sediment at the jetty, observation during site visit

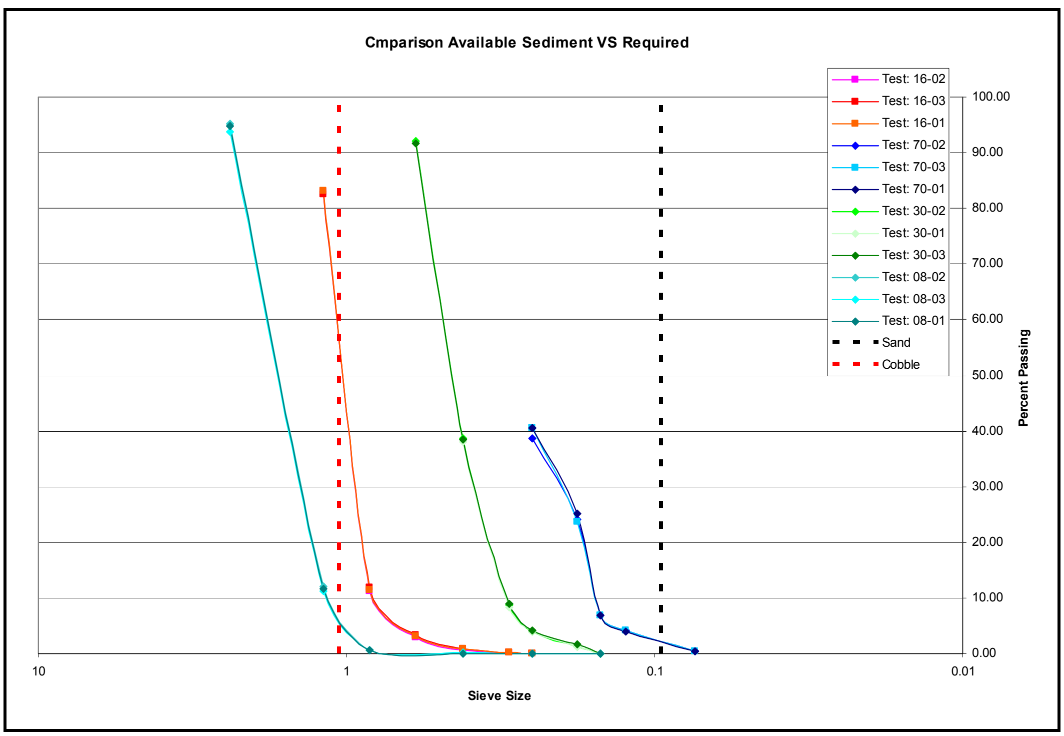

Figure 5.10 Available to WRL Sediment Grain Size distributions VS Prototype Components

Figure 5.11 Photo of sediment placed at the south jetty (prior model runs)

5.4.2. Modeling Alternatives

Existing conditions and four different alternatives of jetty modifications were modeled in the tank[10]. Numbering of alternatives for numerical model corresponds to numbering of prototype alternatives discussed in Section 3 of the report. A description of the alternatives in modeling scale terms follows.

Alternative 1A Alternative 1A consists of a jetty extension approximately 600ft in length along the existing alignment. The extension was constructed of armor rock at a 1V:1.5H slope to a length of 24.66 ft to the angle, with a height of 0.38 ft above the SWL. Figure 5.12 shows a photograph of the Alternative 1A jetty.

[10] Please note that a total of 10 alternatives were analyzed during hydrodynamic modeling. However, to reduce physical modeling effort only four alternatives were selected for testing in the wave tank. See Section Alternatives)

Figure 5.12 Alternative 1A jetty extension.

Alternative 3A - Alternative 3 consists of a jetty extension with two alignments: alignment 1 – along the existing jetty alignment approximately 14.7 ft (200ft prototype) in length; Alignment 2 – jetty extension along at the angle of 150° azimuth approximately 29.4 ft (400 ft) in length. Jetty extension segments are constructed of armor rock at a 1V:1.5 H slope.

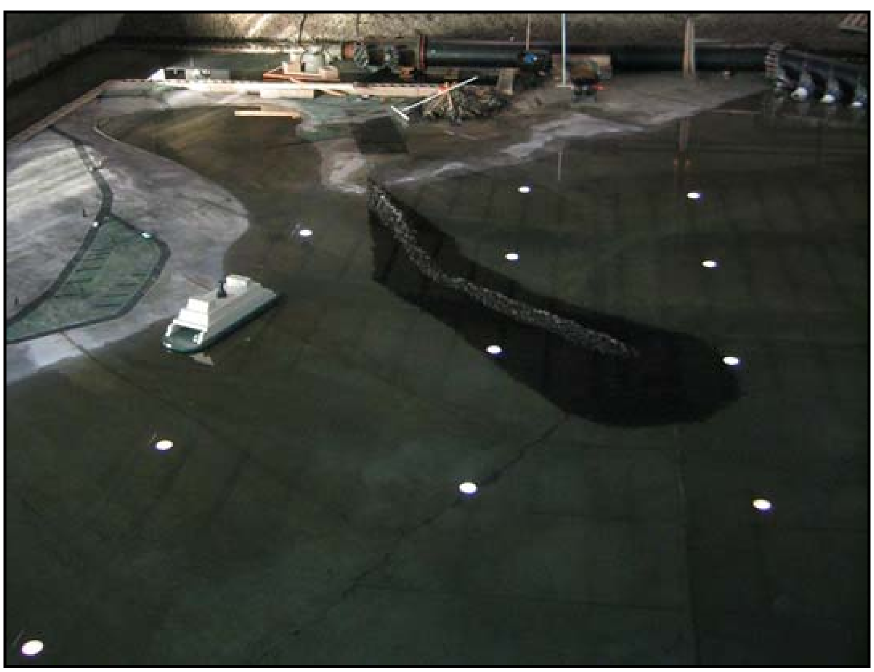

Figure 5.13 Alternative 3A jetty extension with 2 alignments.

Alternative 3B -Submerged jetty extension with two alignments: Alignment 1 – jetty extension along the existing jetty alignment approximately 200 ft in length; Alignment 2 – submerged jetty extension along at the angle of 150° azimuth approximately 400 ft in length. Jetty extension segments are constructed of armor rock at a 1V:2H slope to a length of 14.66 ft to the angle, 10 ft from the angle. The jetty extension is submerged to a depth of 0.13 ft below MSL. The modeled rock size used was 54.5 mm (model-scale). This is the same modeled rock size used in the construction of the existing jetty. Figure 5.14 shows a photograph of the Alternative 3B jetty.

Figure 5.14 Alternative 3B submerged jetty extension with 2 alignments.

Alternative 4

Jetty extension with two alignments: Alignment 1 – along the existing jetty alignment approximately 200 ft in length; Alignment 2 – at the angle of 150° azimuth approximately 400 feet in length. Jetty extension along both alignments is a wave barrier. The wave barrier covers upper two-thirds of the water column. Figure 5.15 is a photograph of the Alternative 4, Alignment 2 wave barrier.

Figure 5.15 Alternative 4, Alignment 2 wave barrier

5.5. Wave Generation and Measurement

5.5.1. Wave Generation

Waves were generated with a fully programmable multi-panel piston-type generator controlled by the GEDAP™ wavemaker control program. The testing program consists of spectral waves with a TMA spectral shape. The wave spectra consisted of both unidirectional and wave spectra with a cosine-squared type spreading function to represent directional wave spectra.

Due to a limitation of the wavemaker, the wave conditions in the basin were determined by averaging the output of the three wave gauges. This methodology has the potential to cause the incident wave height at any particular gauge to be 40% higher than the desired modeled wave height. Small wave disturbances, i.e., reflection, can appear to be larger due to the method of wave creation.

As it was mentioned above in Section… wave climate at Keystone Harbor is complex and constitutes with different sources, including Pacific Ocean waves, wind waves generated in Straight Juan de Fuca, and wind waves generated by local winds in Admiralty Bay. Also, the analysis above (See Section 5.2) shows that local wind waves is the major contributing factor to shoreline erosion to the east of the jetty. Therefore the wind waves of this sources (local winds) were selected for physical modeling. Table 5.3 shows the wave parameters to generate in both prototype and model scale. Prototype wave parameters correspond to exacerbated extreme wave parameters hindcasted for the project area (See Section 2). Exacerbation was done to improve sensitivity of the model to measure potential effects from the jetty alternatives on shoreline erosion. All wave conditions were modeled for two wave headings, 195^o and 210^o from true North.

Table 5.3 Prototype and model wave parameters.

Prototype Scale		Model Scale	
$H_{s,p}$ [ft]	$T_{p,p}$ [s]	$H_{s,m}$ [ft]	$T_{p,m}$ [s]
3	4	0.08	0.63
4	5	0.10	0.79

Each wave storm on the model was run for a duration of 20 minutes. A 20 minute duration was chosen to assure appearance of a measurable effect from the alternatives on the shoreline. Due to the length of the model, wave reflection in the basin was of particular concern. To minimize wave reflection, horsehair was placed along the manifold and its adjacent wall. Two systems of wave were used for numerical modeling: irregular and monochromatic. The irregular waves were generated to a TMA spectrum with a 3.3 groupiness factor (γ). A 3.3 groupiness factor was selected. A directional spreading function was not utilized because the wavelength required to obtain a true spreading in the model basin is greater than the wavelength being modeled. As a result, the spreading function has no effect on the wave being modeled and the modeling results.

5.5.2. Wave Measurement

Waves were measured with 3 surface piercing resistance-type wave gages. The locations of the wave gages for the monochromatic and irregular wave conditions are in shown in Figure 5.15 and Figure 5.16, respectively. The measurements were conducted at each node of the grids showing on the figures. Duration of the wave record was: for monochromatic wave - 1-minute, irregular waves - 3-minutes. To keep the same total duration of measurements (20 minutes per run) and considering the difference in duration of wave record, the measuring grid for irregular waves is widely spaced and has less measuring points. .

Three gages were mounted on a frame suspended from the bridge that spans the basin. The gages were mounted to the frame at varying intervals. Multiple locations of the wave gages were utilized to determine the diffraction pattern around each alternative.

A wave direction calibration was performed after the installation of the wavemaker. The wave direction was measured by photographing the wave field from overhead and digitally extracting the wave angle. These results were compared to the values input to the generation system as a check.

The gages calibrated over a range of 20 cm, in 2 cm increments, using a linear calibration curve. The approximate error of the calibration is submillimeter, resulting in an overall accuracy of wave measurement of ±1 mm. The free-surface elevation data was acquired at a rate of 50 Hz and is filtered at 10 Hz.

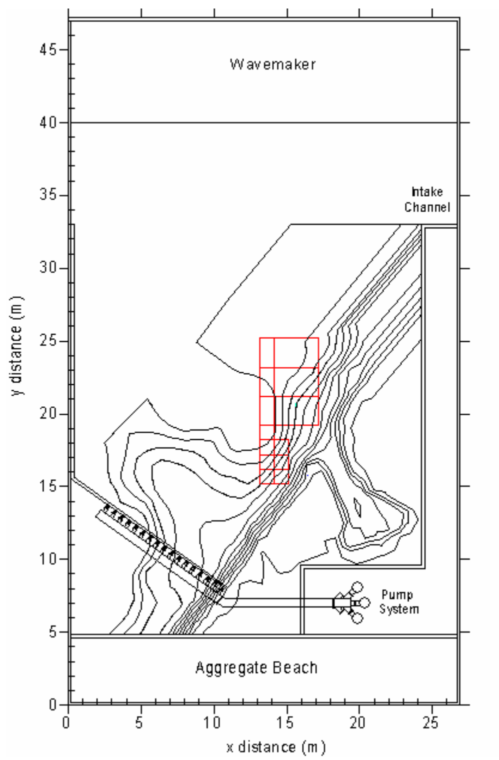

Figure 5.15 Monochrome wave data measuring grid

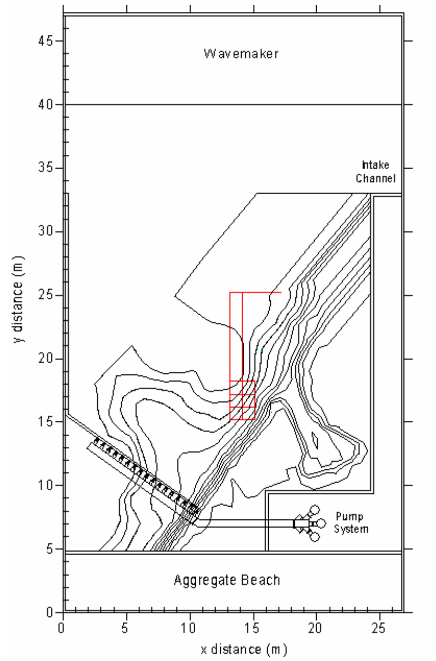

Figure 5.16 Irregular wave data measuring grid

5.6. Current Generation and Measurement

5.6.1. Current Generation

The currents were generated with 2 BIBO BS-2250 pumps, rated at 5000 GPM each. The water was pumped to an outflow manifold to the east of the project site. The outflow manifold has 17 variable flow ports. The flow port located closest to the model was not utilized because its location placed it above the still water level. The flowrate from each port was controlled independently through a manual gate valve. The current velocity in the model was controlled by adjusting the flow out the manual gate valves. This was required because the pumps are not variable speed.
A required uniform flow of 19 cm/sec (See Section 5.3.1 Table 5.2 above) in the model basin was achieved by stepping through multiple stages that verified the current speed. Additional modification to the model was conducted to achieve the required current velocities on the model. These modifications included:

- Installation of current guide wall at the end of the outflow manifold. The guide has minimized the gyres and took away the viscous effects between still and moving water. Preliminary analysis and observation on the model showed that the wall effects on the modeling results appeared to be minor.
- Lifting of a sheet metal plate (approximately 0.032 mm in width), located along the last 5 flow ports. The sheet metal plate was placed along the last 5 flow ports to act as a lip between the manifold and model contours.

The currents were adjusted to obtain the desired velocity at steady-state. It was observed in both the visual and measurement stages that when the pump is first turned on it creates a seiche in the basin. Seiches observed in the model were also visible in the channel. To determine the length of time between starting the pump and the time that the flow reached a steady state (i.e. the seiche was no longer visible) a calibration test was run for a duration of 2 hours. Analysis of this data determined that a steady state velocity in the basin occurs after approximately 5-10 minutes. Further analysis of the steady state velocity showed that a 2% difference in recorded current velocities existed between a 3-minute and 10-minute averaging period.

.

Figure 5.17 Photograph of the current outflow manifold.

5.6.2. Current Measurement

The currents were measured using 3-D ADV (Acoustic Doppler Velocimeters) gages. Four gages were mounted on a frame suspended from the bridge that spans the basin. The gages were mounted to the frame at 1 meter intervals. The ADV was setup with x-axes positive onshore (parallel to the side wall), y-axes positive perpendicular to the side wall and z-axes positive up. The current grid–points measuring locations are shown in Figure 5.18. Figure 5.19 shows a photograph of the ADV gages mounted on the bridge and ready to be submerged for measurement.

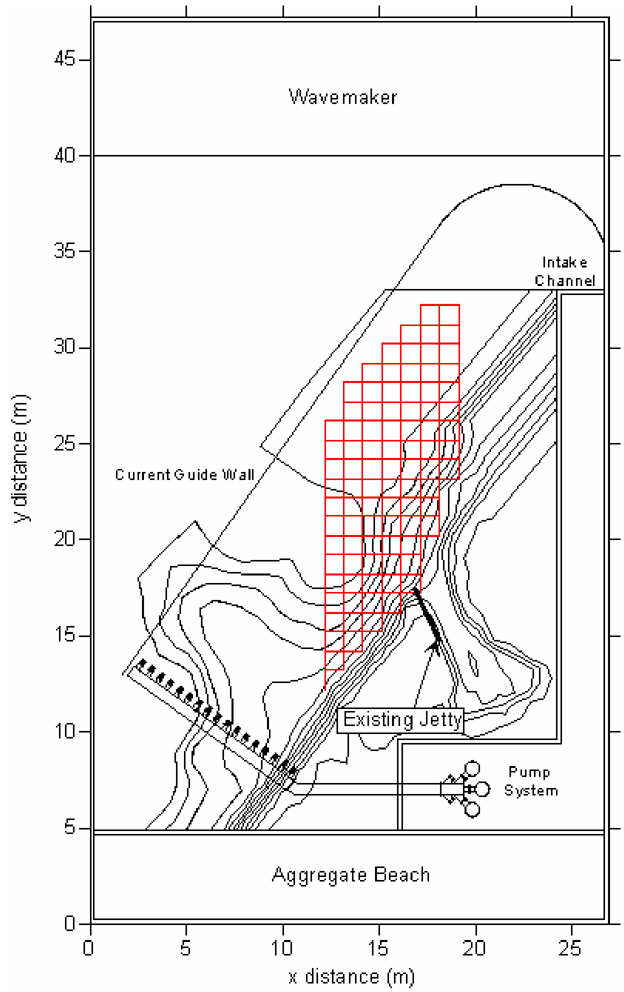

Figure 5.18 Current measurement grid

Currents were measured (recorded) for 3 minutes at each location with frequency of 1Hz. The current meters were set-up to measure the depth-averaged velocities.

Figure 5.19 Photograph of the ADV gages mounted on the bridge ready for measurement.

5.7. Shoreline Change Analysis

The shoreline changes at the upper part of the beach were measured on the physical model. Measurements were conducted during the wave modeling runs by taken images of the beach at the following intervals: 1-minute, 10-minute, and 20-minute. In addition, the measurements of location of the edge of the water were conducted at the beginning and end of each run.

5.8. Model Testing Scheme

The model testing scheme consisted of separate runs for waves and for currents. The following Table shows the list of the modeling runs, characteristics of main modeling parameters and stage of completion for each of the run.

Table 5.4 Model Testing Scheme

Run Number	Jetty Configuration	Current Speed (cm/sec)	Wave Type	Wave Height (mm)	Wave Period (sec)	Wave Heading from True N
1	Existing Jetty	19	-	-	-	-
2	Alternative 1a Jetty	19	-	-	-	-
3	Alternative 3b Jetty	19	-	-	-	-
4	Alternative 3a Jetty	19	-	-	-	-
5	Alternative 4 Jetty	19	-	-	-	-
6	Existing Jetty	-	Monochromatic	2.3	0.63	195
7	Existing Jetty	-	Monochromatic	2.3	0.63	210
8	Existing Jetty	-	Monochromatic	3.0	0.79	195
9	Existing Jetty	-	Monochromatic	3.0	0.79	210
10	Existing Jetty	-	Irregular	2.3	0.63	195
11	Existing Jetty	-	Irregular	3.0	0.79	195
12	Alternative 1a Jetty	-	Monochromatic	2.3	0.63	195
13	Alternative 1a Jetty	-	Monochromatic	2.3	0.63	210
14	Alternative 1a Jetty	-	Monochromatic	3.0	0.79	195
15	Alternative 1a Jetty	-	Monochromatic	3.0	0.79	210
16	Alternative 1a Jetty	-	Irregular	2.3	0.63	195
17	Alternative 1a Jetty	-	Irregular	3.0	0.79	195
18	Alternative 3a Jetty	-	Monochromatic	2.3	0.63	195
19	Alternative 3a Jetty	-	Monochromatic	2.3	0.63	210
20	Alternative 3a Jetty	-	Monochromatic	3.0	0.79	195
21	Alternative 3a Jetty	-	Monochromatic	3.0	0.79	210
22	Alternative 3a Jetty	-	Irregular	2.3	0.63	195
23	Alternative 3a Jetty	-	Irregular	3.0	0.79	195
24	Alternative 3b Jetty	-	Monochromatic	2.3	0.63	195
25	Alternative 3b Jetty	-	Monochromatic	2.3	0.63	210
26	Alternative 3b Jetty	-	Monochromatic	3.0	0.79	195
27	Alternative 3b Jetty	-	Monochromatic	3.0	0.79	210
28	Alternative 3b Jetty	-	Irregular	2.3	0.63	195
29	Alternative 3b Jetty	-	Irregular	3.0	0.79	195
30	Alternative 4 Jetty	-	Monochromatic	2.3	0.63	195
31	Alternative 4 Jetty	-	Monochromatic	2.3	0.63	210
32	Alternative 4 Jetty	-	Monochromatic	3.0	0.79	195
33	Alternative 4 Jetty	-	Monochromatic	3.0	0.79	210
34	Alternative 4 Jetty	-	Irregular	2.3	0.63	195
35	Alternative 4 Jetty	-	Irregular	3.0	0.79	195

5.9. Modeling Results

5.9.1. Cross Channel Currents Modeling Results

Current measurements, collected during physical modeling were processed and analyzed to develop the basis for validation of the computer model and evaluate the modeling alternatives (four alternatives) with regard to their performance.

5.9.1.1 Computer Model Validation

5.9.2. Cross Channel Currents Modeling Results

Current measurements, collected during physical modeling were processed and analyzed to developed the basis for validation of the computer model and evaluate the modeling alternatives (four alternatives) with regard to their performance.

5.9.1.1 Computer Model Validation with Physical Model

Computer modeling was conducted to simulate the current velocities measured in the physical model. It should be noted that the physical model simulates steady ebb current conditions. Therefore, a steady flow current model with the ADCIRC computer models was constructed with the same geometry and boundary conditions as in the physical model. The ADCIRC grid and bathymetry for the existing conditions are shown in Figure 5.20 (a) and (b), respectively.

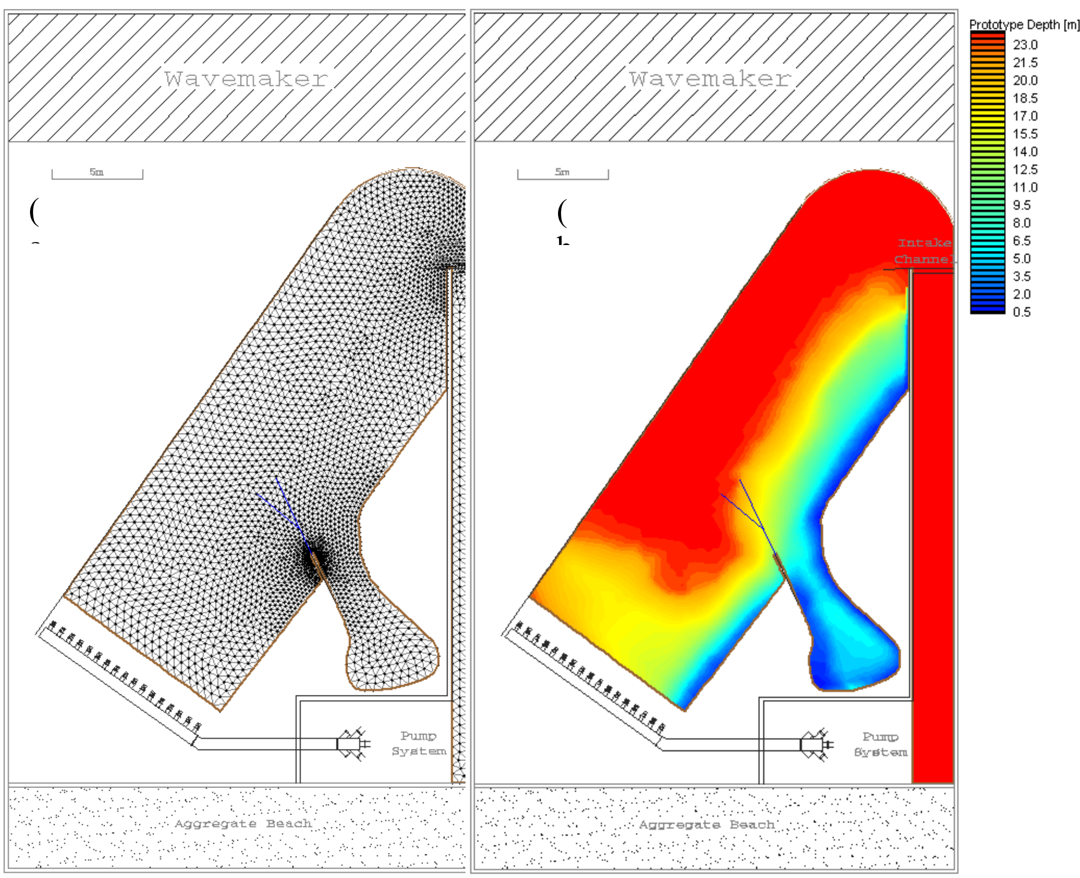

Figure 5.20 (a) Modeling grid and (b) bathymetry of the ADCIRC model of the Physical modeling domain.

The ADCIRC model's input boundary condition was a constant normal flow-rate along the boundary coincident with the input manifold to reproduce the conditions in the physical model. The bathymetry of the physical was obtained from the LIDAR measurement as discussed in Section 5.4.1. All geometrical dimensions, velocities, and flowrates were computed in Prototype scale in the ADCIRC model. The model was run until a steady-state solution was achieved. The existing conditions as well as Alternative 1a, 3a, 3b, and 4 were run in the ADCIRC physical model simulation.

To compare the ADCIRC simulation and the measured currents from the physical model, the current magnitudes were extracted along the channel centerline as described in Section 5.9.2 for both the ADCIRC and the physical model results. The magnitudes are plotted as a function of distance from the Keystone Pier in Figure 5.21. In this Figure, the results from the ADCIRC simulation are shown with lines and the physical model results are shown with points.

The existing conditions were reproduced quite well, only slightly over-predicting the velocity in the far offshore at the modeling domain boundary. These over-predictions likely result from the modeling domain boundary effects such that an unsteady inflow rate of water from the manifold (farthest offshore ports on the manifold might be less than nearshore ports due to friction losses in the manifold). The inconsistency between modeling and physical results at the boundaries of domain does not impact model accuracy at the project area. Specifically that ADIRC modeling domain boundaries specified far away from the project area and do not effect on the modeling results at the project.

Reasonable correlation between computed and measured at physical model velocities were obtained for Alternative 1A and Alternative 3A.

The larger differences between computer and physical modeling results are observed for submerged jetty and wave barrier jetty – Alternatives 3B and 4. The difference is mostly because these alternatives produce three-dimensional flow cases while ADCIRC computes depth-averaged flow rather than resolving variation of velocity over depth. This is evident from the results shown in Figure 5.21. For Alternative 3B, the ADCIRC simulation predicts almost no difference from the existing conditions while the physical model shows flow patterns similar to Alternative 3A. The results for the wave barrier – Alternative 4 – are predicted well in the lee of the wave barrier and under-predict the velocity offshore of the wave barrier. These differences would result in a conservative estimate of both of the Alternative's ability to reduce velocities in the navigation channel, and would underestimate the change in velocity at the tip of the jetties.

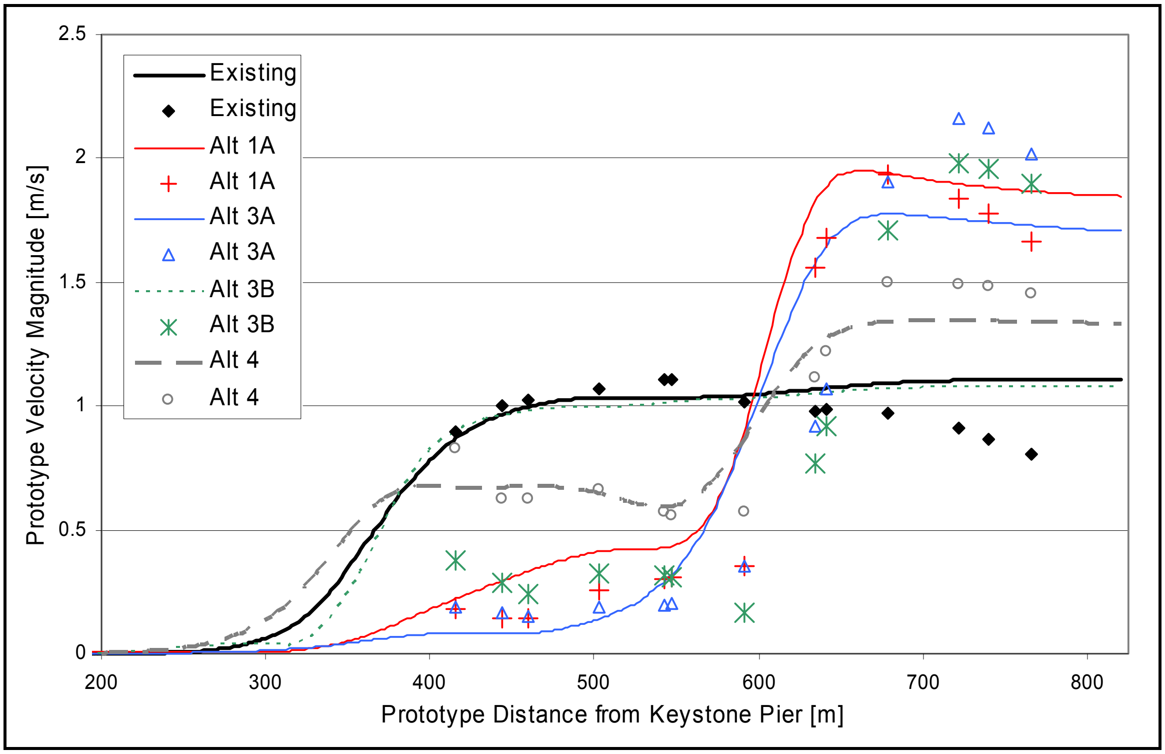

Figure 5.21 Comparison of the physical modeled velocities (points) and the ADCIRC simulated velocities (lines) along the Keystone Channel as a function of distance from the Keystone Pier. All values are in prototype scale.

Some representative flow fields for the physical model and the simulation of the physical model by ADCIRC are shown in Figures 5.22 (a) and (b) and 5.23 (a) and (b). The Current magnitudes (color contours) and directions (arrows) are superimposed on the model tank schematic. Figure 5.22 shows the existing conditions for (a) measured in the physical model and (b) simulated with ADCIRC. The magnitudes and directions are quite similar, and the distribution of velocity over the entire measured area matches well. Note that only the region where measurements were made in the physical model is shown for both the physical model and the ADCIRC simulation. Figure 5.23 shows the currents for Alternative 4 (the wave barrier) (a) measured in the physical model and (b) simulated with ADCIRC. Here, the overall flow pattern is similar for the physical model and the ADCIRC, but the velocity magnitudes differ by approximately 0.25m/s (about 15% difference) in the offshore. This is likely the inability of ADCIRC to accurately simulate three-dimensional flows that occur around the wave barrier, which blocks the upper two-thirds of the water column. The ADCIRC simulation does not correctly block the flow and less water passes around the end as compared to the physical model.

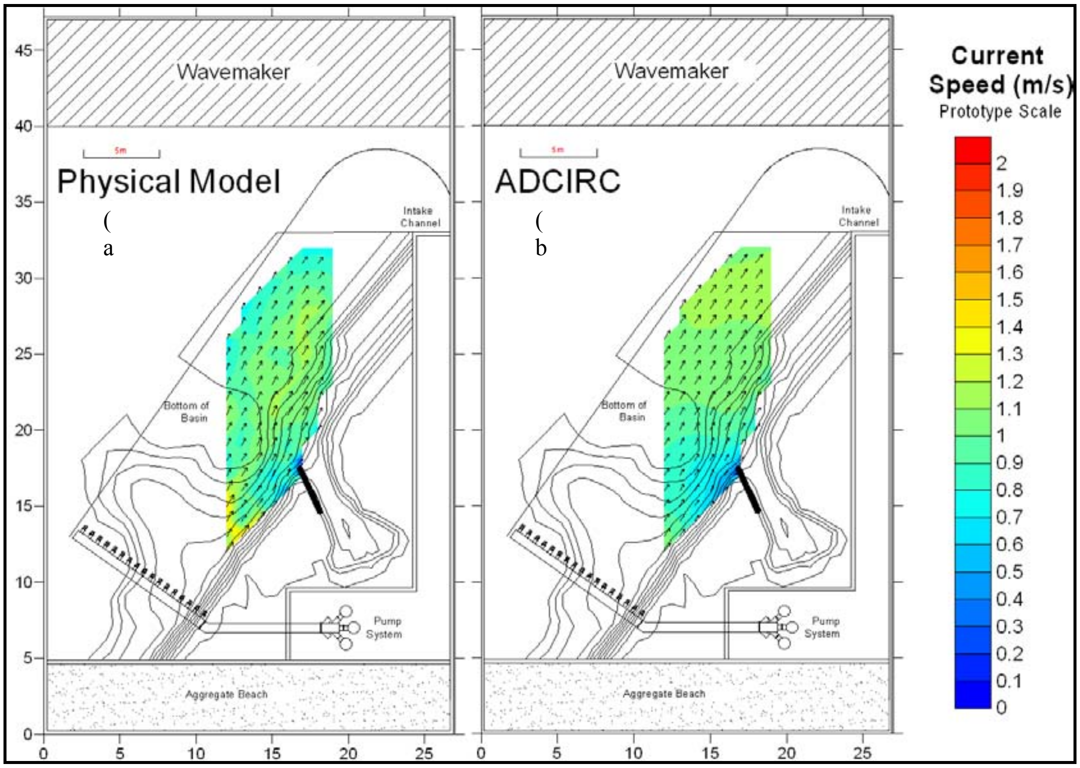

Figure 5.22 **Current velocity and direction for existing conditions for (a) physical model and (b) ADCIRC simulation of the physical modeling domain.**

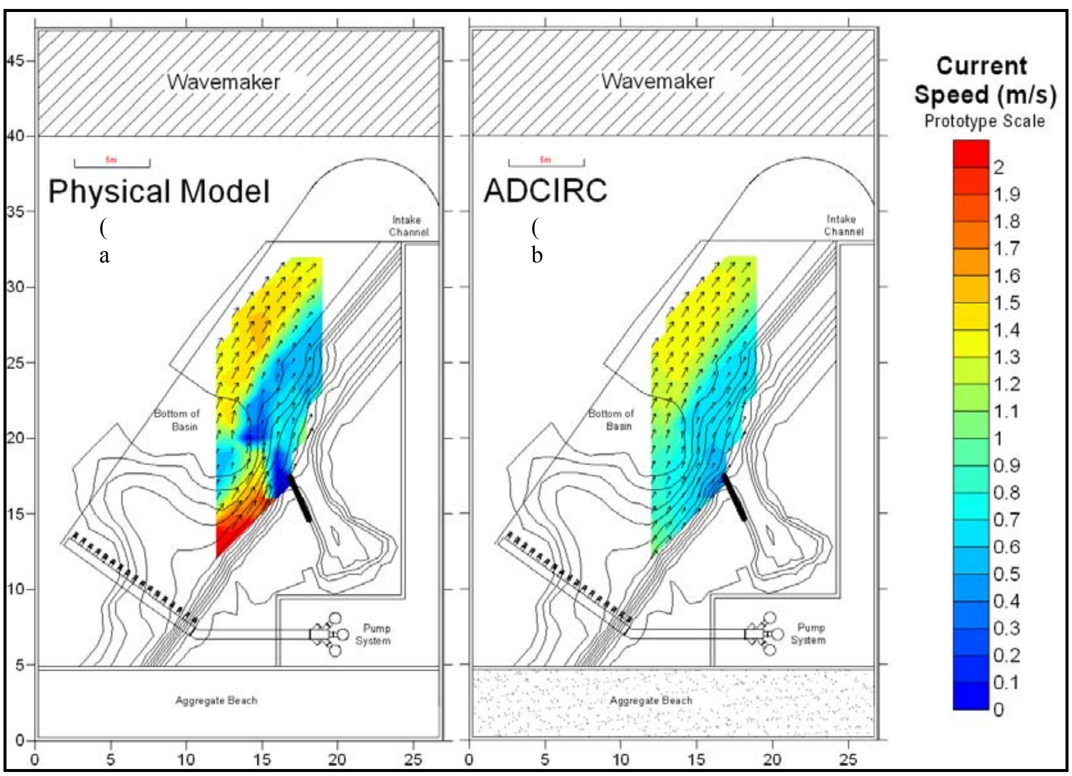

Figure 5.23 **Current velocity and direction for Alternative 4 for (a) physical model and (b) ADCIRC simulation of the physical modeling domain.**

5.9.3. Alternative Comparison

Performance of each alternative was evaluated with regard to reductions channel cross-current velocities relatively to the existing conditions. Two sets of data: spatial distribution of current velocities and direction, covering the area of the entrance of Keystone Harbor and controlling grid-point measurements of current velocities and directions were developed from the modeling results and are used for evaluating of the alternatives. Figure 5.24 and 5.25 shows the examples of plan-view current velocities and direction for existing conditions and Alternative 1A. Similar figures for all modeling alternatives are presented in Appendix B.

Evaluation was conducted by comparing spatial distribution of current velocities in the footprint and along the centerline of the channel.

Comparison of spatial velocities distribution was conducted by subtracting post-project spatial velocities distributions from existing conditions spatial distributions and zooming the results in the footprint of the channel. Figures 5.26-5.29 show the difference of current velocities spatial distributions between the existing and Alternative conditions. Comparison of centerline velocities was conducted by subtracting post-project velocities from existing conditions velocities along the channel centerline. Figure 5.26 shows the current speed for the existing and Alternative conditions along the channel centerline. Figure 5.31 shows the line of difference in current velocities. The evaluation was based on the distance from the terminal seaward to the location in the channel where the alternatives provide lower cross current speeds than existing conditions. The distance corresponding reduction of current velocities was calculated from model outputs and the results are tabulated in Table 5.5.

Table 5.5 Current Velocity Reduction VS Potential Distance for Deceleration

Alternative	Average Velocity along Channel Centerline (m/s)	Distance of Reduced Currents Measured from Existing Jetty South End (m)
Existing Conditions	1.00	-
Alternative 1a	0.18	246.1
Alternative 3a	0.17	289.8
Alternative 3b	0.31	296.4
Alternative 4	0.68	246.1

Summarizing the discussion above we have concluded the following:

- Physical model validates the numerical model ADCIRC for existing conditions of Keystone Harbor.
- Observation from physical modeling supports the conclusions from Section 4.1 report on results of numerical modeling for Alternatives 1, 3, 3B, and 4.
- It is shown that all four jetty extension alternatives tend to reduce the average velocity along the navigation channel, indicating they would all benefit navigation in general.
- The differences in alignment of the jetty extension alternative do not appear to have a significant effect on the length of channel over which currents are reduced compared to existing conditions.
- Although Alternative 3B appears to reduce current speeds along the larger distance, the speed reduction along most of this distance is small and may not be significant for navigation enhancement.

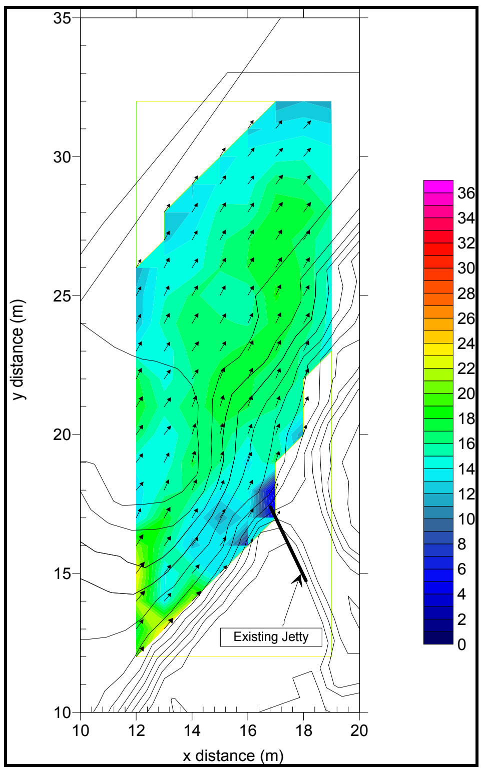

Figure 5.24 Existing conditions current results

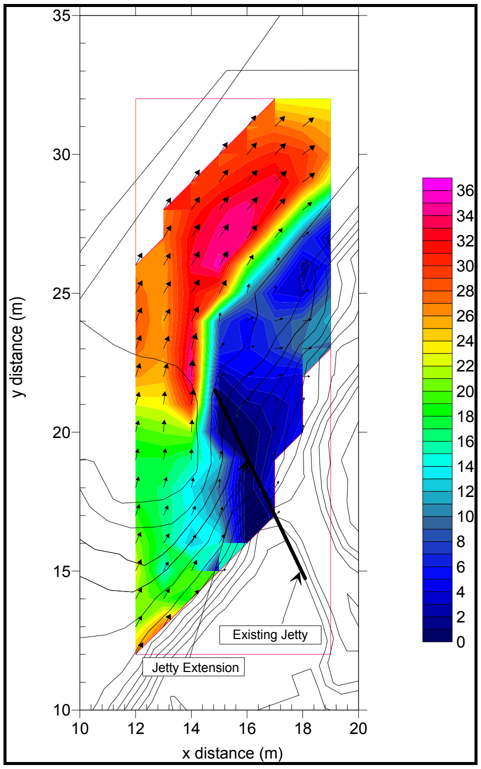

Figure 5.25 Alternative 1A current results

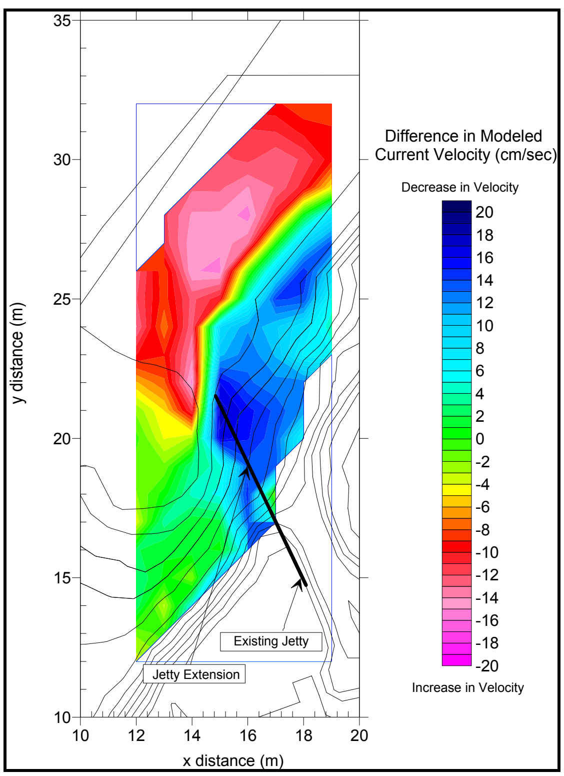

Figure 5.26 Current velocity difference – Existing – Alternative 1

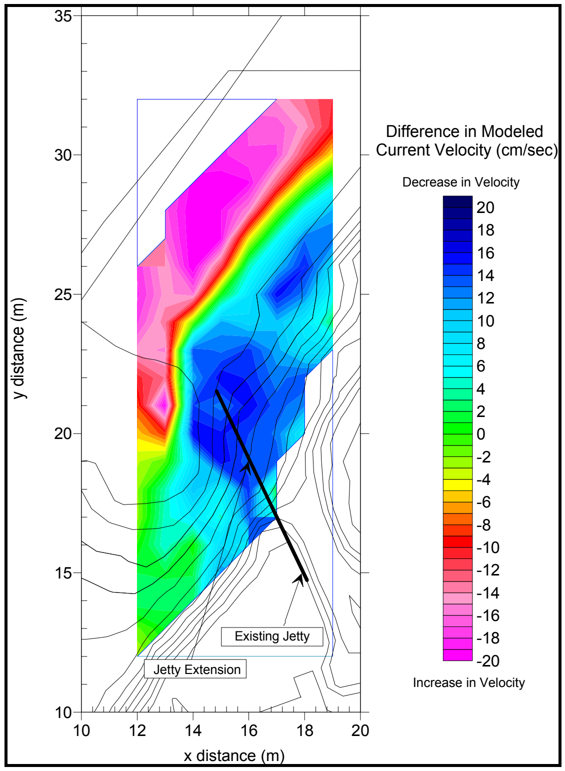

Figure 5.27 Current velocity difference – Existing – Alternative 3a

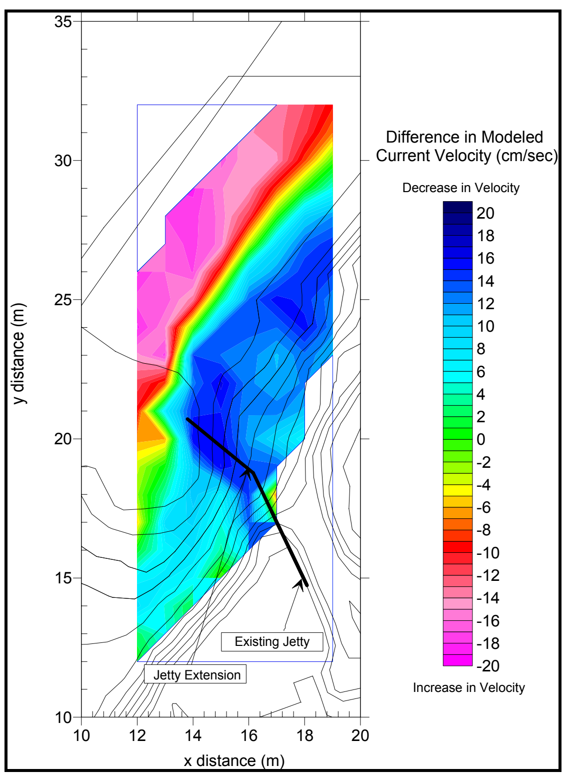

Figure 5.28 Current velocity difference – Existing – Alternative 3b

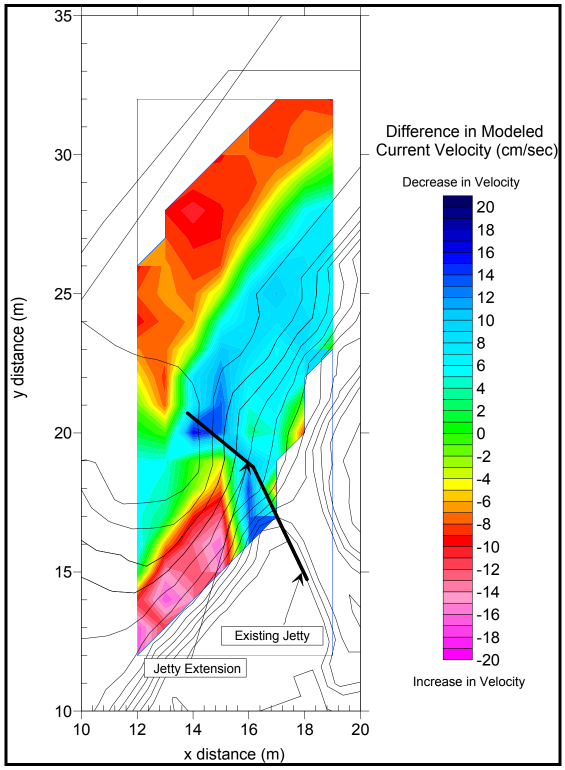

Figure 5.29 Current velocity difference – Existing – Alternative 4

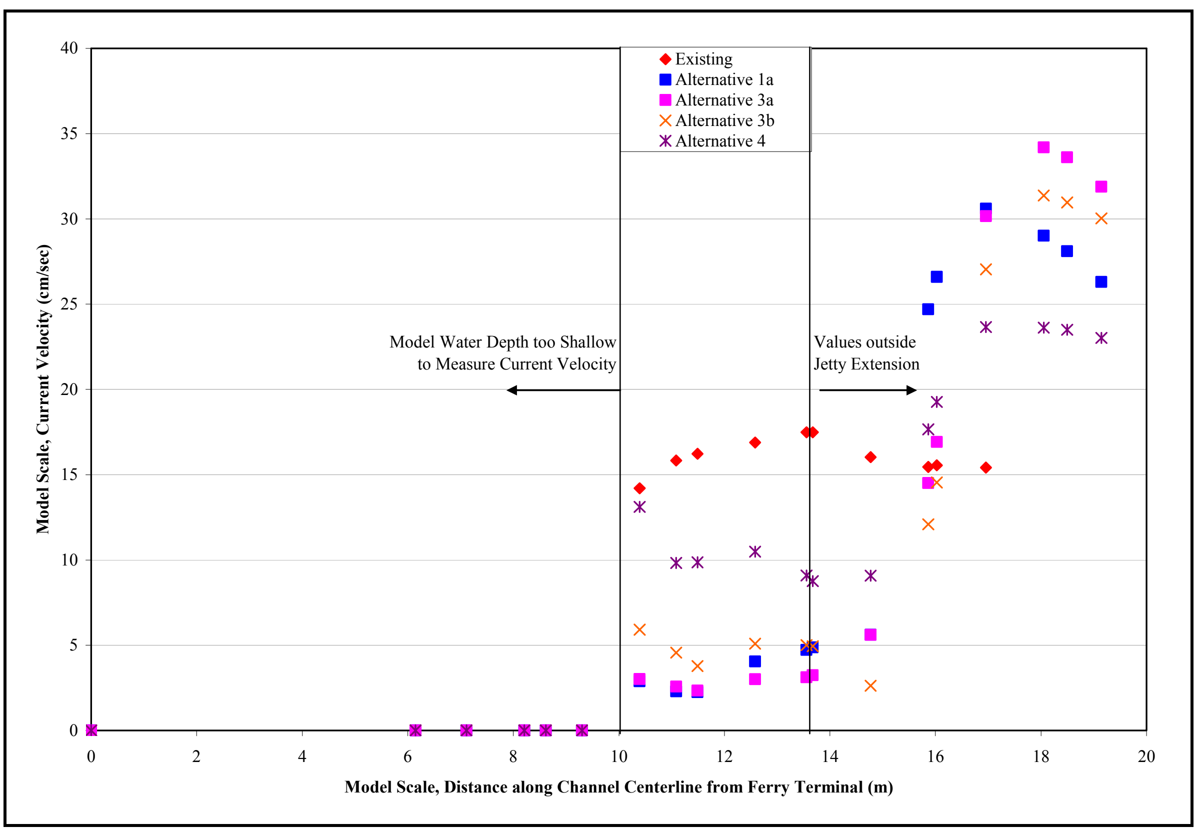

Figure 5.30 Modeled current velocities along channel centerline for existing and Alternative conditions

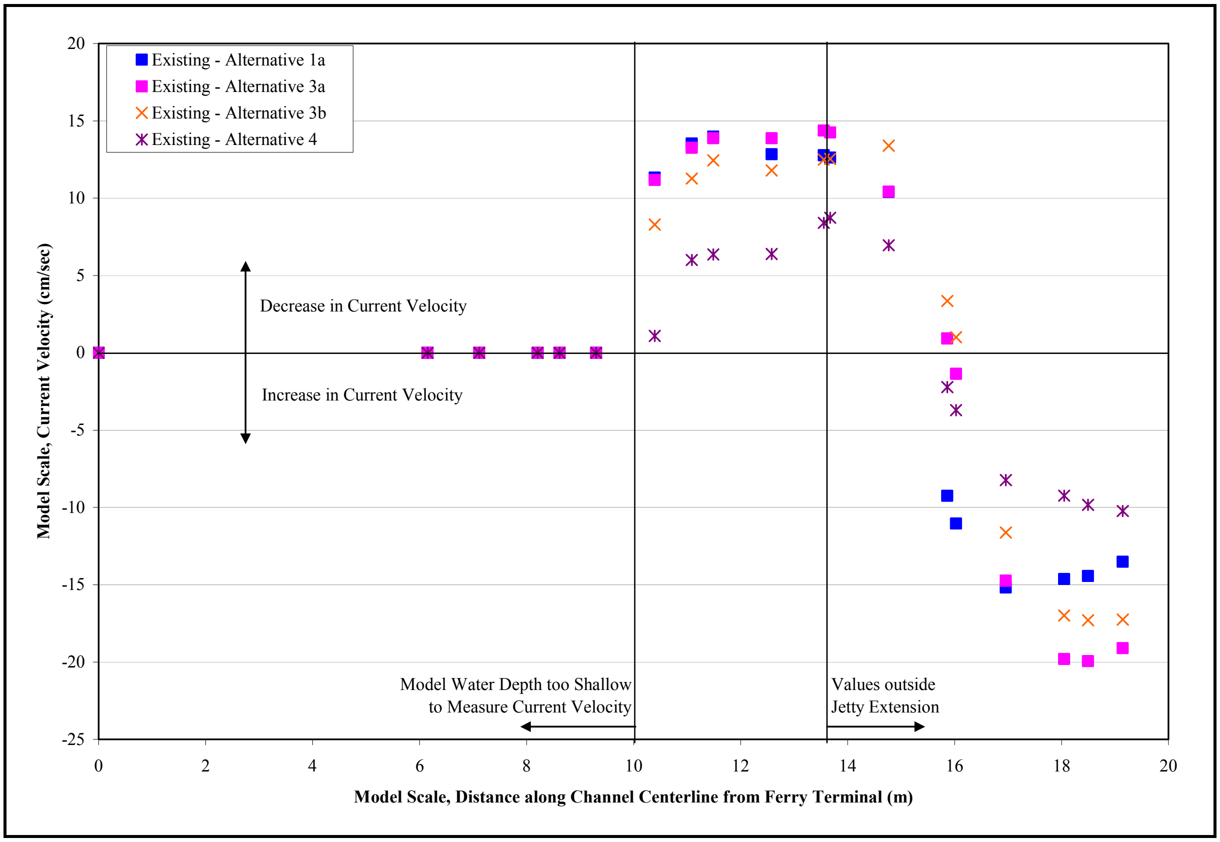

Figure 5.31 Difference in modeled current velocities along channel centerline for existing and Alternative conditions

5.10. Wave Refraction/Diffraction and Shoreline Change Modeling Results

Wave measurements, collected during the physical modeling was processed to obtained three sets of data: plan distribution of wave heights and periods over the modeling domain, controlling grid-points wave height and wave period data, and shoreline changes relatively to existing conditions data. Table 5.7 defines the wave conditions modeled for each alternative as a case number. Figure 5.32 and Figure 5.33 show the example of wave height distribution on the modeling domain for existing conditions and for Alternative 1A respectively, for Case 6. The modeled wave conditions around the jetty extension were not plotted due to the scarcity of data around it. Plotting this data would provide an unrealistic representation of the waves around the jetty extension. Figure5.34 shows wave heights for all modeling scenarios and for all alternatives at model grid location 15, 15 (See Figure 5.15). Based on this figure, it is evident that there exist peaks in the measured wave conditions for the existing and alternative cases. These peaks are attributed to the methodology of obtaining the desired basin wave height (see Section 5.5.1). Figures 5.35 to 5.38 show the change of shoreline position for Alternative 1A during Case 6. A complete discussion of the change in shoreline position is done in Section 5.11. The figures similar to that from the above examples but for all alternatives are presented in the Appendix.

Table 5.7 Case number definitions

	Wave Type	Wave Height (mm)	Wave Period (sec)	Wave Heading from True N
Case 1	Monochromatic	2.3	0.63	195
Case 2	Monochromatic	2.3	0.63	210
Case 3	Monochromatic	3.0	0.79	195
Case 4	Monochromatic	3.0	0.79	210
Case 5	Irregular	2.3	0.63	195
Case 6	Irregular	3.0	0.79	195

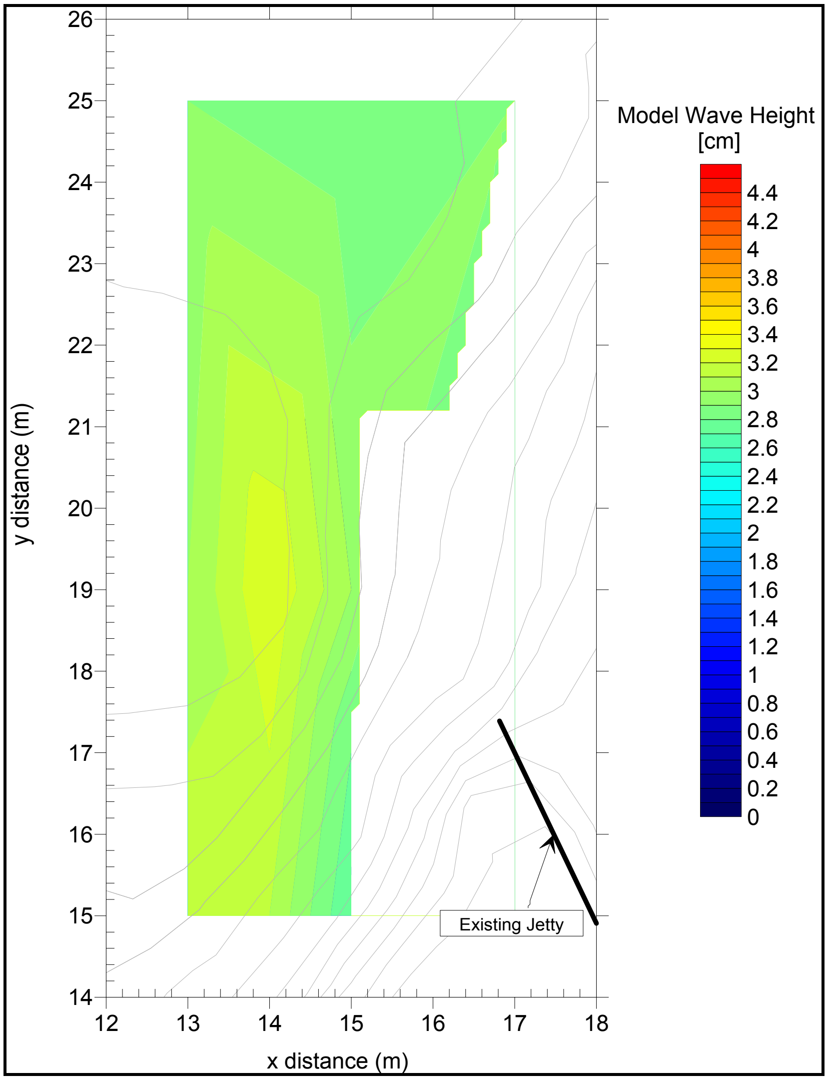

Figure 5.32 Wave Refraction/Diffraction modeling results for Existing conditions Case 6

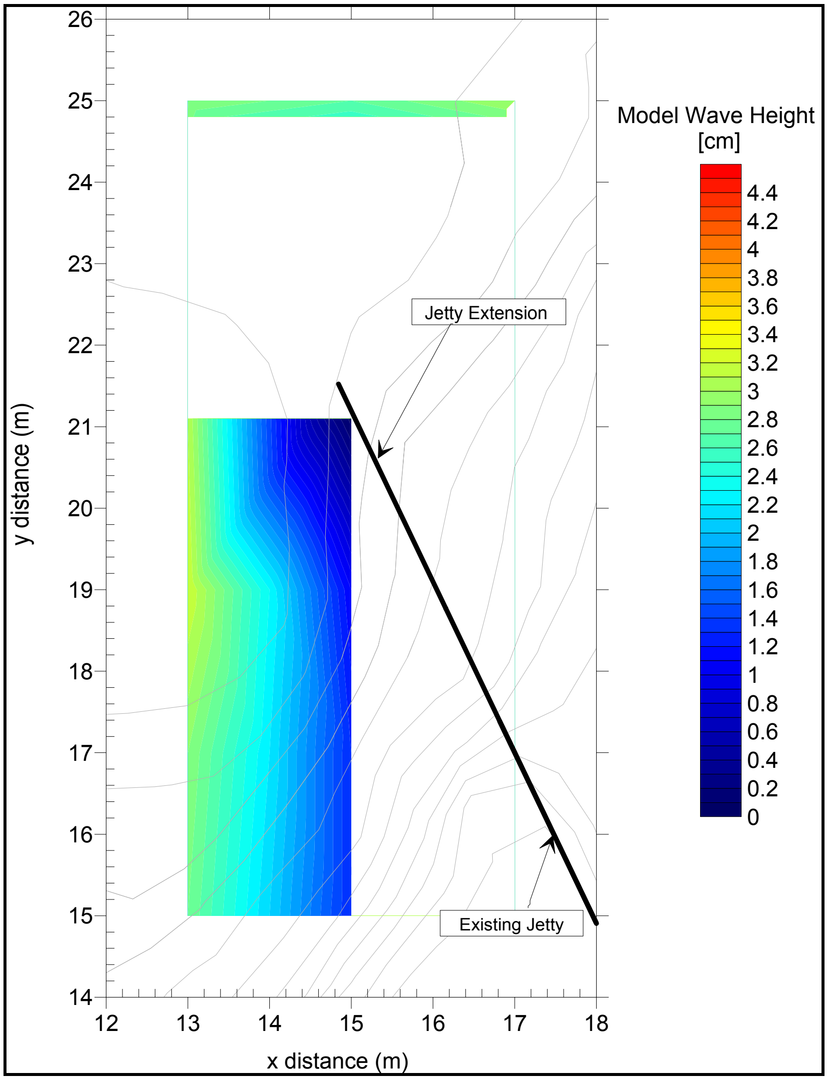

Figure 5.33 Wave Refraction/Diffraction modeling results for Alternative 1A Case 6

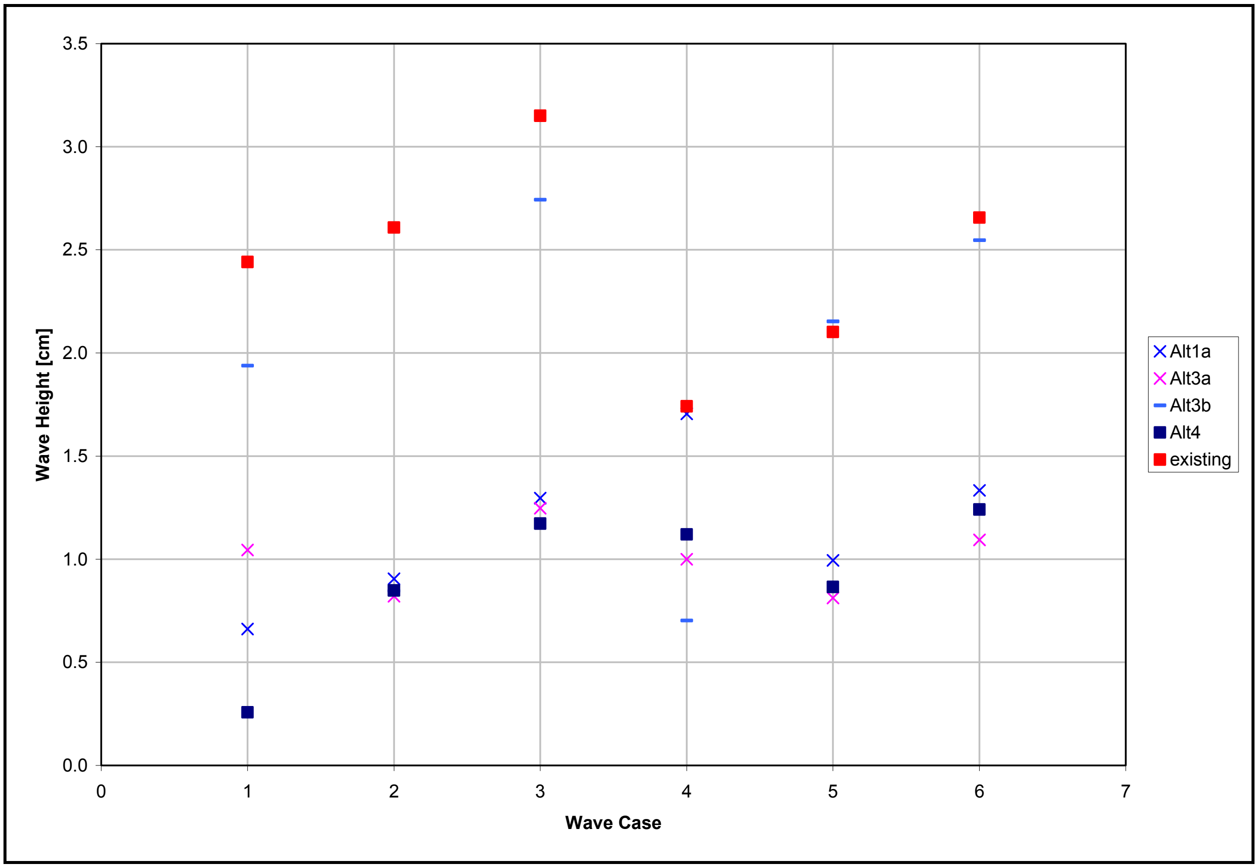

Figure 5.34 Wave Refraction/Diffraction modeling results for all alternatives and cases at model grid location 15, 15

Figure 5.35 Shoreline change analysis for Alterative 1A, Case 6 time = 0 minutes

Figure 5.36 Shoreline change analysis for Alterative 1A, Case 6 time = 10 minutes

Figure 5.37 Shoreline change analysis for Alterative 1A, Case 6 time = 15 minutes

Figure 5.38 Shoreline change analysis for Alterative 1A, Case 6 time = 20 minutes

The visual observations of the shoreline change for the existing conditions and four testing alternatives is presented in below.

Table 5.9 Existing conditions shoreline change qualitative summary

Existing Conditions	Sediment Sorting	Small submerged bar	Berm	Beach cusp in swash zone	Scarp	Additional notes
Case 1	Yes	formed 5-10cm from beach face	none	none	none	beach visibly less protected from incident waves
Case 2	Yes	formed 5-10cm from beach face	none	none	none	beach visibly less protected from incident waves
Case 3	Yes	Offshore location increases with distance from jetty (10-30cm)	none	early formation of individual cusps that increase with distance from the jetty	none	Substantial sediment transport on and offshore, away from jetty. Eroded approx. $30cm^3$ of sediment, exposing concrete at 1.8 m from jetty.
Case 4	Yes	Offshore location increases with distance from jetty (10-30cm)	none	early formation of individual cusps that increase with distance from the jetty	none	Higher sediment transport rates and volumetric changes than all other conditions
Case 5	Minor	none	none	none	none	Trivial sediment transport
Case 6	Minor	none	none	none	none	Trivial sediment transport

Table 5.10 Alternative 1a shoreline change qualitative summary

Alternative 1a	Sediment Sorting	Small submerged bar	Berm	Beach cusp in swash zone	Scarp	Additional notes
Case 1	Minor	None	none	none	none	Waves visibly larger inside jetty compared to Alt3a and Alt3b
Case 2	Minor	None	none	none	none	Waves visibly larger inside jetty compared to Alt3a and Alt3b
Case 3	Yes	formed as large cusps merge	none	individual cusps that increase with distance from the jetty, with the larger measuring 10cm in width and 25 cm in length	none	Cusps and berms made predominately from fine materials
Case 4	Yes	formed as large cusps merge	none	individual cusps that increase with distance from the jetty, with the larger measuring 10cm in width and 25 cm in length	none	Cusps and berms made predominately from fine materials
Case 5	Minor	None	none	none	none	Finer material placed higher on beach face
Case 6	Minor	None	none	none	none	Finer material placed higher on beach face

Table 5.11 Alternative 3a shoreline change qualitative summary

Alternative 3a	Sediment Sorting	Small submerged bar	Berm	Beach cusp in swash zone	Scarp	Additional notes
Case 1	None	early formation at 22 cm offshore	none	none	none	shadow zone in lee of jetty shows little to no reflection
Case 2	Scarp formed primarily of cobble, visible separation from fine sediments located at toe of scarp	None	none	small beach cusps, smaller beach cusps merge to form uniform scarp	Approx. 0.5 cm high on back shore.	
Case 3	Finer materials higher on beach face. Offshore bar built primarily with cobble sized material approx. 25 cm from the beach/water interface	berm formed along beach face, approx. 10 cm in width and 0.5 cm in height	none	early formation, dominant boundaries further away from jetty shadow zone	none	waves visibly larger in lee of jetty than were observed in Case 1 and 2
Case 4	finer materials higher on beach face	None	none	beach cusps formed	beach cusps merge into uniform scarp	minor accretion at jetty toe
Case 5	Fine materials dominate swash zone with larger cobbles below and above swash zone	minimal berm formation in shadow zone of breakwater	none	less prominent than monochromatic cases	much steeper than monochromatic cases	
Case 6	Fine materials dominate swash zone with larger cobbles below and above swash zone	minimal berm formation in shadow zone of breakwater	none	less prominent than monochromatic cases	much steeper than monochromatic cases	

Table 5.12 Alternative 3b shoreline change qualitative summary

Alternative 3b	Sediment Sorting	Small submerged bar	Berm	Beach cusp in swash zone	Scarp	Additional notes
Case 1	Yes	None	none	early beach cusp formation	early beach cusps merge into scarp	shadow zone significantly less than Alt3a, more wave energy transmitted across submerged berm, slight offshore migration of sediment near jetty toe
Case 2	Yes	None	none	individual cusps that increase with distance from the jetty	none	
Case 3	Berm predominately small grain materials, bar predominately large grain materials	Approximately 30 cm beach	nearshore berm formed within first 10 min at 1.6m from the jetty, berm width as large as 15 cm.	none	none	
Case 4	Berm predominately small grain materials, bar predominately large grain materials	Approximately 30 cm beach, minimal offshore bar formation	berm formed further from jetty than in Case 3, berm width as large as 15 cm and 0.5 cm in height; berm not uniform	individual cusps	none	
Case 5	Minor	None	none	none	none	
Case 6	Minor	None	none	none	none	

Table 5.13 Alternative 4 shoreline change qualitative summary

Alternative 4	Sediment Sorting	Small submerged bar	Berm	Beach cusp in swash zone	Scarp	Additional notes
Case 1	Minor	None	none	small beach cusp	none	noticeably larger waves in the harbor channel compared to rubble mound structure cases, minimal sediment movement
Case 2	Minor	None	none	small beach cusp	none	noticeably larger waves in the harbor channel compared to rubble mound structure cases, minimal sediment movement, reflected wave noticeable
Case 3	upper face of cusps comprised mostly of fine grains	None	none	beach cusps 5 cm wide and 30 cm long forming furthest from the jetty	none	
Case 4	upper face of cusps comprised mostly of fine grains	None	none	beach cusps 5 cm wide and 30 cm long forming furthest from the jetty	none	
Case 5	Minor	None	none	none	none	
Case 6	Minor	None	none	none	none	

5.11. Physical Modeling Summary

- Physical model of current velocities validates the numerical model ADCIRC for existing conditions of the Keystone Harbor and four tested alternatives .Observation from physical modeling supports the conclusions from Section 4.1 report on results of numerical modeling for Alternatives 1, 3, 3B, and 4. Consequently, the results of numerical modeling for all other alternatives, including Jetty Relocation Alternative, should be considered verified. Physical model has provided the means for adjustments the results from 2-Dimensional numerical model to three-dimensional processes observed on Alternatives 3B and 4.
- Physical model of wave refraction diffraction validated the conclusions from COASTOX numerical modeling for existing conditions and tested alternatives.
- Physical model of shoreline changes shows that jetty extension alternative will not effect on shoreline erosion. No new trends for the shoreline to the east from the jetty is expected for any of the tested alternatives.

6. Discussion, Summary, and Conclusions

The feasibility study has been initiated by WSF to develop and evaluate the alternatives for improving navigation conditions at the entrance to the harbor. The Hydrodynamic and Physical Modeling Study were conducted to evaluate the jetty modifications alternatives using computer and hydraulic modeling. The proposed alternatives were evaluated based on the comparison of the project controlling factors before construction (known and modeled) for existing conditions with those estimated (modeled) for each of the proposed alternatives (if they were to be constructed). The evaluation of the alternatives was conducted based on the following controlling factors:

- Cross current velocities
- Wave conditions in the channel and in the harbor
- Channel sedimentation and maintenance dredging requirements
- Shoreline erosion Shoreline erosion and bottom scour
- Water quality.

Existing physical conditions at the Keystone Harbor including tides, winds, waves, current velocities, sediment transport (littoral drift) and morphology were evaluated and described using available information and new field data. Major findings from these efforts included input data and boundary conditions for numerical and physical modeling and also defining specific site conditions at the Keystone harbor as follows:

- The wave climate at Keystone Harbor and the adjacent areas is complicated and consists of at least three major wave systems: Pacific Ocean waves, wind storms from the Strait of Juan de Fuca, and local Admiralty Bay wind-waves. Each wave system was described in the study in terms of wave heights, wave periods, and occurrences.
- Several prominent geographic features impact tidal flows in the area and cause significant eddying and flow variability in the tidal currents on both ebb and flood tide. Current data show prevailing flow in a southwesterly ebb direction the majority of time regardless of whether the current is ebbing (flowing northward out of Admiralty Inlet) or flooding (flowing southward into Admiralty Inlet It is possible that construction of the jetty in 1948 has contributed to phenomena of eddy formations at the entrance to the harbor and as a result increase in cross channel currents. The current velocities are of homogeneous current speeds and directions over the water column and allow use of depth-averaged numerical models for describing the flow fields in Admiralty Inlet
- Since the time of constructing the navigation project in 1948 the sediment transport system at the Keystone shoreline has consisted of two littoral cells: East Cell, eastward from the jetty, and West Cell, westward from the navigation channel. These two littoral cells are separated by the jetty and navigation channel. The cells have a limited exchange of sediment between them except for the artificial bypassing associated with the dredging events.
- The sediment transport of coarse material in the vicinity of the project has been predominantly from west to east. Prior construction of the jetty in 1948, small amounts of coarse material had contributed to the westward littoral drift. Since construction of the jetty, the only fine sediment (sand and silt) are transported to the west around the jetty and accumulates in the channel. If construction of the jetty in 1948 was to reduce maintenance dredging, it appears that location of the jetty on the east side of the channel was not the optimal solution.
- Equilibrium and accreting beaches are observed to the east from the jetty at approximately 2 miles along the shoreline (excluding several hundreds feet adjacent to the jetty). It appears that practice of the Corps of Engineers, including placement of dredged material at the east site of the jetty has contributed to changes in shoreline trend from erosional pattern to relative equilibrium pattern.
- The north-south shoreline (west of the ferry terminal) is subjected to continuous retreat and erosion. The retreat rate within one-half mile of the harbor entrance is about 1 ft per year.

Validated and calibrated with three different methods, ADCIRC computer modeling was conducted to investigate tidal currents in the vicinity of the project and to compare the proposed alternatives with regard to ability of reduction the cross current in the channel and effect on physical environments.

- All jetty extension alternatives (excluding Alternative 3B) would reduce significantly cross current velocities in the channel along the length of the extended jetty at approximately 600 ft along the channel. Reduction of cross current velocities along this stretch of the channel is calculated at approximately 70-80% relative to existing conditions.
- Alternative 3B would provide smaller reduction of cross current velocities in the channel along the extended jetty. This reduction would not exceed 30% relative to existing conditions.
- All jetty extension alternatives would provide some reduction of current velocities seaward of the extended jetty at approximately to 20-30% relative to existing conditions. However, prior to reduction, an increase of velocities may occur immediately at the end of the extended jetty. This immediate increase would increase the shear, specifically for Alternatives 1, 1A, 2 and 2A.
- Jetty Relocation Alternative would provide >30-50% reduction of cross current velocities at distance of more than 2,000 ft of the seaward end of the existing jetty.
- Jetty Relocation Alternative would reduce the shear effect at the entrance of the harbor.
- It is likely that construction of the existing jetty in 1948 has changed the pattern of eddies, exacerbating cross-current velocities at the entrance to the harbor. It is likely that prior jetty construction, the strength of the currents was smaller and/or was reduced gradually toward the entrance in accordance to the natural depth. All the jetty modification alternatives would change (to different extent) the tidal flow eddying patterns near the project area. Jetty relocation alternatives would partially re-construct the pre-1948 current patterns at the entrance to the harbor.
- From the perspective of cross current velocities controlling factor, two jetty modifications alternatives Alternative 3 (or 3A) –Rock Dogleg and Alternative 5 – Jetty Relocation are most feasible. These alternatives are recommended for further analysis.

Numerical wind-wave refraction/diffraction/reflection modeling was undertaken to evaluate the effect of the proposed jetty alternatives on the local wave environment surrounding Keystone Harbor. The modeling results are also were used in evaluation of potential effects from alternatives on coastal processes, specifically shoreline erosion, bottom scour, and channel sedimentation. In order to account for complexity of natural wave transformation conditions and optimize the modeling procedure, computer modeling was performed using two different 2-Dimenional computer models, STWAVE and COASTOX. It was found:

- All jetty extension alternatives would reduce wave heights approaching from SE in the channel located along the extended jetty. All jetty extension alternatives (if no deepening and widening channel occurs) would not alter wave conditions in the harbor relative to existing conditions.

- Relocation of the jetty (Alternative 5) would not change waves in the channel to any significance that may effect the navigation for storms approaching from SE and SW
- Relocation of the jetty (Alternative 5) would increase wave heights inside of the harbor relative to existing conditions for both storm conditions, SE and SW. Maximum increase of wave height during extreme storm event, approaching from SE is estimated at approximately 2.0 ft. Increase of wave heights in the harbor is a result mostly of deepening and widening the channel that allows more wave energy penetrate to the harbor
- All jetty extension alternatives except Alternative 3B would significantly reduce wave energy along the shoreline during wave storms approaching from SW.
- Relocation of the jetty (Alternative 5) would reduce wave energy along the shoreline during wave storms approaching from SW
- In the cases of both rock jetties and vertical wall barriers, the composite-alignment solutions seem to provide better protection in the lee side, as they are oriented almost normal to the incident storm wave direction and create significant shadow areas behind the structure.

Sediment transport at Keystone Harbor was evaluated using the LAGRSED two-dimensional sediment transport model. The basis of selecting this model was its unique ability to simultaneously simulate transport of different types of sediment that constitute to the project littoral system. The findings from sediment transport modeling are as follows:

- Under existing conditions sand and finer material (silt, clay) erode from the dredged material placement area and accumulate in the channel and inside the harbor. Small accumulation of sand occurs in the vicinity of the jetty from the east side.
- Erosion under existing conditions occurs in deep water seaward of the harbor entrance. This result appears to be consistent with the general bathymetry of the area. The bottom depression located seaward of the jetty was observed in all available hydrographic surveys. Though this depression is slightly offset from the entrance in the Eastward direction, the general trend of scour shown in the model (depth of scour and orientation of the scour) indicates a similarity between the modeling results and typically observed field processes.
- Implementation of most alternatives (excluding Alternatives 2 and 3B) will not result in increasing existing conditions of sedimentation and maintenance dredging requirements. Furthermore the alternatives may result in a slight reduction of sedimentation in the channel and harbor due to reduction of westward transport of fine sediment.
- Alternative 2 may result in increase of sedimentation in the channel due to high gradient of bottom velocities due to the clearance under the structures.

Sedimentation may include deposition of fine sediment as well as coarse sand and gravel.

- Alternative 3B may result in slightly increase of sedimentation due to the overflow of suspended sediment over the submerged portion of the jetty. Because sedimentation would consists of suspended sediment that is sand and finer material, the added volume of sedimentation would be insignificant.
- Implementation of most alternatives (excluding Alternatives 3, 3B and 4) will not result in loss of sand from the beach to the east of the jetty. Alternatives 3, 3B. and 4 may result in slight removal of sand relative to existing conditions due to increase of the tidal velocities at local spots on the beach. However, this will not effect gravel and coarse sand sediment.

Water quality modeling was conducted using the two-dimensional, finite element hydrodynamic model RMA4 (US Army Corps of Engineers) to evaluate the effect of the proposed jetty construction alternatives on water quality in the Keystone Harbor. The findings from the modeling are as follows:

- Evaluation of the alternatives with regard to the water quality criteria was based on computation residence time for each of the alternative.
- Results of computation show that most of jetty extension, including Alternative 3 may increase slightly the residence time. It implies only slight deterioration of the water quality. Observation on the model shows that quality of the water in the harbor (residence time) is inversely proportional to blockage by the jetty the upper water column area.
- Alternative 3B would reduce slightly the residence time in the harbor. This effect is due to the jetty submergeness. Lowering the jetty crest elevation would reduce residence time and improve the water quality in the harbor if requires reduction of water.
- Alternative 5, Jetty Relocation would reduce the residence time and most likely improve the water quality in the harbor.

High resolution scale physical modeling of the Keystone Ferry Terminal and the jetty extension alternatives was conducted in the three-dimensional wave basin at Oregon State University to meet the following goals:

- Verify and validate the numerical modeling of tidal flow circulation and wave transformation
- Qualitatively evaluate the alternatives with regard to their effect on shoreline changes in the vicinity of the jetty
- Quantitatively assess the effects of the jetty extensions and relocations on cross-channel currents at the entrance to Keystone Harbor

7. References (for computer simulation section)

Cannon, G.A., Ebbesmeyer, C.C., and Bruce J. Nairn (2001) "Circulation Variations in the Northern Main Basin of Puget Sound", Puget Sound Research 2001, 15p.

Cannon, G.A., Editor (1978) "Circulation in the Strait of Juan de Fuca: Some Recent Oceanographic Observations", NOAA Technical Report ERL 399-PMEL 29, 49p.

CH2MHill, Inc. (2003) "Keystone Ferry Terminal Relocation Feasibility Study", Prepared for Washington State Ferries, August 2003, 148p.

Coastal and Hydraulics Laboratory (2001) "Users Guide to RMA4 WES Version 4.5", U.S. Army Corps of Engineers Research and Development Center, Waterways Experiment Station, May 2001.

Holbrook, J.R., Muench, R.D., Kachel, D.G. and C. Wrightn (1980) "Circulation in the Strait of Juan de Fuca: Recent Oceanographic Observations in the Eastern Basin" NOAA Technical Report ERL 412-PMEL 33, Seattle, WA.

Le Provost, C., Genco, M. L., Lyard, F., Vincent, P., and P. Canceil (1994) "Spectroscopy of the World Ocean Tides from a Finite Element Hydrological Model", J. Geophysical Research, 99, 24777–24798.

Luettich, R.A., Jr., Westerink, J.J., and N.W. Scheffner (1992) "ADCIRC: an advanced three-dimensional circulation model for shelves coasts and estuaries, report 1: theory and methodology of ADCIRC-2DDI and ADCIRC-3DL" Dredging Research Program Technical Report DRP-92-6, U.S. Army Engineers Waterways Experiment Station, Vicksburg, MS, 137p.

Luettich, R.A., Jr. and J.J. Westerink (1999) "Implementation of Bridge Pilings in the ADCIRC Hydrodynamic Model: Upgrade and Documentation for ADCIRC Version 34.19", Contractors Report, department of the Army, US Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS, November 19, 1999, 8 p.

Maderich, V.S., Brovchenko, I., Fenical, S., and V. Shepsis (2004, in print) "Lagrangian Sediment Transport Model" Submitted to American Geophysical Union Fall Meeting December, 2004.

Shepsis, V. and D. Simpson (2000) "Propwash Modeling for Confined Aquatic Disposal" Presented at Conference on Dredged Material Management: Options and Environmental Considerations, Boston, MS, December 6-9, 2000.

Smith, J.M., Sherlock, A.R. and D.T. Resio (2001) “STWAVE: Steady-State Spectral Wave Model, User’s Manual for STWAVE”, CHL SR-01-1, Version 3.0, Coastal and Hydraulics Laboratory, USACE.

Yarnell, D.L. (1934a) “Pile Trestles as Channel Obstructions”, U.S. Dept of Agriculture, Tech. Bull. 429.

Yarnell, D.L. (1934b) “Bridge Piers as Channel Obstructions”, U.S. Dept of Agriculture, Tech. Bull. 442.

Hughes, S. (1993) Physical Models and Laboratory Techniques in Coastal Engineering. The Advanced Series on Ocean Engineering, Vol. 7, World Scientific, SINGAPORE.

CH2M HILL 2002. Keystone, WA Ferry Terminal Feasibility Study: Winds, Waves, and Currents. Prepared by Ken Lilly, Jr., CH2M HILL, 27 August 2002.

Hallermeier, Robert J. 1983. Sand Transport Limits in Coastal Structure Design. Proceedings, Coastal Structures ’83 Conference. American Society of Civil Engineers, New York, NY, p. 703 – 716.

Keuler, Ralph 1988. Map Showing Coastal Erosion, Sediment Supply, and Longshore Transport in the Port Townsend 30-Minute Quadrangle. Folio of the Port Townsend Quadrangle, Miscellaneous Investigation Series, Map I-1198-E.

Leenknecht, David, Andy Szuwalski, and Ann Sherlock 1992. Automated Coastal Engineering System version 1.07e. Coastal Engineering Research Center, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Seattle District Corps of Engineers 1972. Keystone Harbor Shoaling Study Technical Narrative. Inclosure 1 to Disposition Form dated 1 August 1972 from Chief, Engineering Division to Chief Operations Division.

CH2M Hill (2003) “Keystone Ferry Terminal Relocation Feasibility Study”, Prepared for Washington State Ferries, August 2003, 148p.

Coastal and Hydraulics Laboratory (2001) “Users Guide to RMA4 WES Version 4.5”, U.S. Army Corps of Engineers Research and Development Center, Waterways Experiment Station, May 2001.

Holbrook, J.R., Muench, R.D., Kachel, D.G. and C. Wright (1980) “Circulation in the Strait of Juan de Fuca: Recent Oceanographic Observations in the Eastern Basin” NOAA Technical Report ERL 412-PMEL 33, Seattle, WA.

Le Provost, C., Genco, M. L., Lyard, F., Vincent, P., and P. Canceil (1994) “Spectroscopy of the World Ocean Tides from a Finite Element Hydrological Model”, J. Geophysical Research, 99, 24777–24798.

Luettich, R.A., Jr., J.J. Westerink, and N.W. Scheffner (1992) “ADCIRC: an advanced three-dimensional circulation model for shelves coasts and estuaries, report 1: theory and methodology of ADCIRC-2DDI and ADCIRC-3DL” Dredging Research Program Technical Report DRP-92-6, U.S. Army Engineers Waterways Experiment Station, Vicksburg, MS, 137p.

Luettich, R.A., Jr. and J.J. Westerink (1999) “Implementation of Bridge Pilings in the ADCIRC Hydrodynamic Model: Upgrade and Documentation for ADCIRC Version 34.19”, Contractors Report, department of the Army, US Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS, November 19, 1999, 8 p.

Maderich, V.S., Brovchenko, I., Fenical, S. and V. Shepsis (2004, in press) “Lagrangian Sediment Transport Model” Submitted to American Geophysical Union Fall Meeting December, 2004.

Shepsis, V. and D.P. Simpson (2003) “Propwash Modeling for Confined Aquatic Disposal. Proceedings of Conference on Dredged Material Management: Options and Environmental Considerations. National Sea Grant College Program, M.I.T., Boston, Massachusetts, December 6-9, 2000.

Smith, J.M., Sherlock, A.R. and D.T. Resio (2001) “STWAVE: Steady-State Spectral Wave Model, User’s Manual for STWAVE”, CHL SR-01-1, Version 3.0, Coastal and Hydraulics Laboratory, USACE.

Yarnell, D.L. (1934a) “Pile Trestles as Channel Obstructions”, U.S. Dept of Agriculture, Tech. Bull. 429.

Yarnell, D.L. (1934b) “Bridge Piers as Channel Obstructions”, U.S. Dept of Agriculture, Tech. Bull. 442.